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Use of the Observational Method to Verify Design of Earth Retention Structures

Timothy H. Bedenis
Soil and Materials Engineers, Inc., Plymouth, MI

Christopher G. Naida
Soil and Materials Engineers, Inc., Plymouth, MI

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Timothy H. Bedenis, P.E.,
Soil and Materials Engineers, Inc.
43980 Plymouth Oaks Blvd.
Plymouth MI, USA, 48170

Christopher G. Naida, P.E.
Soil and Materials Engineers, Inc.
43980 Plymouth Oaks Blvd.
Plymouth MI, USA, 48170

ABSTRACT

Two case histories are presented for earth retention structures where the results of construction monitoring were used to verify key criteria used for the design of the structures. One case history consists of a sheet-pile bulkhead structure for the unloading of bulk aggregate products in Bay City, Michigan. The front wall of the sheet-pile bulkhead was analyzed for heavy surcharge pressures from the adjacent aggregate stockpiles. The sheet-piles are laterally supported by sheet-pile deadman and tierods. Slope indicator tubes were used to monitor lateral movements next to the sheet-piles. The results from the slope indicators were then used to verify the stability of the bulkhead under the heavy surcharge loads from the aggregate stockpiles. A second case history consists of a deep braced excavation for the construction of a processing pit for a new steel rolling mill in Dearborn, Michigan. A very stiff "King Pile" system was used to resist the large lateral pressures below the bottom of the excavation due deep deposits of soft clay soils. Multiple levels of heavy bracing consisting of double rows of heavy steel sections for the walers and large pipe struts were used to support the walls. During construction strain gauges were affixed on selected struts to verify the design strut loads.

INTRODUCTION

Geotechnical engineers design earth retention structures based on well established geo-mechanical engineering principals and practices. These methods are based on both theoretical and empirical techniques backed by many years of research and confirmed with monitoring and measurements of actual earth retention systems as they are being constructed and placed into service.

Due to the inherent variability of soils and the empirical nature of the design methods, the geotechnical engineer should establish methods to verify their earth retention designs. Observational methods allow the engineer to evaluate and adjust their systems in real time as the structures are first loaded. Engineers and contractors often do this simply by visually observing the structures, or with optical survey measurements on accessible points on the surface of the structure, since the movement of the structure is most times the final determination of a successful design. However, for key locations on critical structures various types of instrumentation can provide better insight into the performance of the earth retention system.

Engineers monitoring earth retention structures (as with other practitioners) are often under the impression that with instrumentation, "more is better". Therefore, they specify programs that generate large amounts of data and therefore become too complex to properly collect, evaluate, and analyze in time sufficient to make any difference to the project. Such systems are simply documenting the conditions for future study. This can be very important for academic or legal purposes, but the opportunity to adjust the design and construction base on the information is lost.

In this paper the authors describe two cases where the observational approach used targeted instrumentation programs on key elements of each earth retention system's structure to quickly and efficiently verify the crucial aspects of the design. Such targeted programs provide valuable information regarding the response of the systems. For the case studies presented below, the targeted instrumentation programs varied between about $10,000 to $20,000 in cost. As such, they vary in return on investment as compared to more extensive and comprehensive monitoring systems.

CASE NO. 1: AGGREGATE STORAGE FACILTY

A former bulk oil storage facility and depot on the Saginaw River in Bay City, Michigan was converted to a bulk aggregate storage facility (see Figure 1). The conversion required an existing boat slip to be dredged to allow for large lake freighters to dock and unload bulk aggregate materials. These vessels self-unload their bulk aggregate cargo with swinging conveyor systems to the areas surrounding the boat slip (see Figure 2).

Fig. 1 Aggregate Storage Facility.

The placement of the stockpiles from the conveyor results in large and heavy cone-shaped piles of aggregate relatively close to the slip (see Figure 3). The aggregates are then loaded into trucks and shipped to concrete plants, asphalt plants, or to construction sites on an as needed basis. Figure 4 shows a typical layout of the aggregate stockpiles surrounding the slip. Placing the aggregate stockpiles close to the slip allows the operator to maximum the storage area of the facility and minimizes the amount of material handling. Moving the aggregate stockpiles to other areas of the site, once the materials are unloaded from the freighters, is very costly.

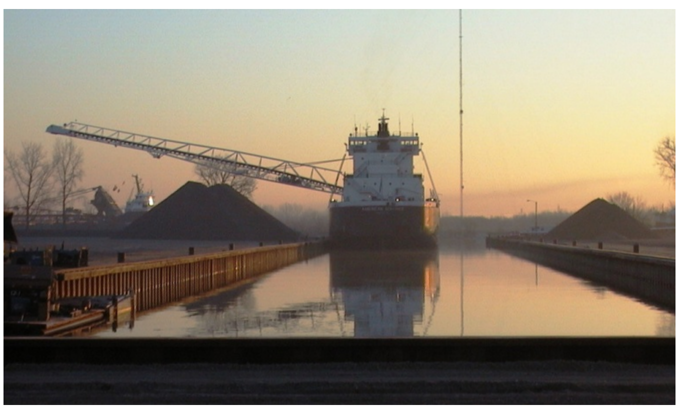

Fig 2. Freighter Unloading Aggregate.

Site and Soil Conditions

The previous boat slip was about 1,500 feet long, extending inland from the river, and about 160 to 180 feet wide. The depth of water in the slip was typically about 10 to 15 feet, but the far west end had filled over the years and was only a few feet deep. The sides of the slip consist of earthen embankments, with only a small pier for docking and connection of piping for pumping of the oil. The area around the slip was relatively flat and clear.

The soil conditions consisted of sand fill and natural alluvial sands over a deep deposit of lacustrine clay. The depth to the top of the clay increased from only a few feet on the west end of the slip to as deep as about 60 feet on the east side of the slip closest to the river. There were various deposits of peat and organic silt trapped between the fill and natural sand, and interbedded within the sand stratum. The frequency of the organic deposits also increased toward the river and ranged from about 3 to 10 feet thick.

Fig. 3 Conical Stockpile

The upper portion of the clay was of a very stiff to hard consistency due to desiccation from when the level of the Great Lakes were much lower than today. The clay was of a stiff to medium consistency below a depth of about 40 feet. A very dense, highly over-consolidated clay till (locally referred to as 'hardpan') was encountered at depth of 80 feet. This till was many feet thick and underlain by sandstone and shale rock formations.

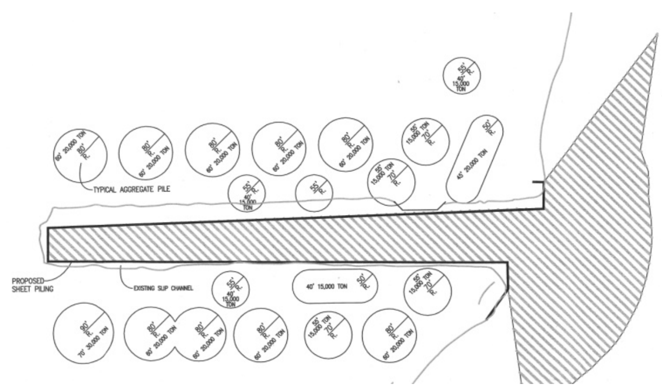

Fig. 4 Aggregate Stockpile Plan.

New Boat Slip Design

The new slip area roughly matched the area of previous one. Such a slip could technically accommodate the largest freighters on the Great Lakes, which are about 1,000 feet long, 100 feet wide, and can carry over 70,000 tons of materials. However, most of the vessels using this facility are somewhat smaller at 600 to 700 feet long, 80 feet wide, with a capacity of about 25,000 tons.

There were two basic problems with the existing slip. The slip had too shallow of a draft and it lacked a bulkhead wall. The previous oil depot was able to pump oil from the shallow draft barges anchored in the middle of the slip. However, the heavily loaded bulk freighters required much deep drafts and needed to dock directly next to the bulkhead wall to allow the unloading conveyors to reach as far back as possible from the slip to provide a second row of stockpiles (see Figure 4).

Required Depth. The design draft of the slip was 26 feet below normal water level. As with all shipping in the Great Lakes region, if the lake level water drops, the volume of cargo in the freighter would be adjusted accordingly to reduce the draft depth of the ship and allow for safe transport in and out of the slip. Therefore, a relatively deep draft of water was required to accommodate low water levels.

Bulkhead Wall. A new steel sheet-pile bulkhead wall was constructed around the perimeter of the existing slip. Most areas required the sheeting to be installed from the existing embankment. A continuous sheet-pile deadman wall connected to steel rods "tierods" was used to laterally support the main bulkhead wall. To control lateral movements, the tierods and deadman had to be installed prior to backfilling behind the bulkhead wall. Figure 5 shows a typical profile of the bulkhead wall system.

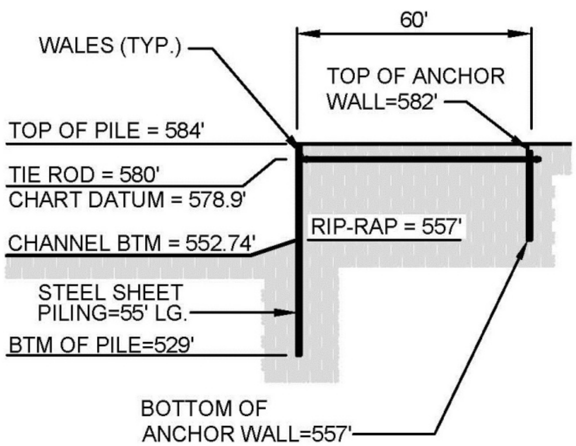

Fig. 5 Typical Steel Sheet-Pile Wall Bulkhead.

As designer of the bulkhead, we had to first determine the lateral earth pressures on the walls. Basically, these lateral earth pressures can be divided into two categories; active and passive earth pressures, and lateral pressures due to surface surcharge loads.

Lateral Soil Pressures. The effective (drained) soil parameters controlled the design of the wall. The active and passive pressures were determined using the log-spiral method Caquot A. & Kerisel, J. [1948]. Refer to Table 1 for the soil parameter and earth pressure coefficients used for the design.

Surcharge Loads. The main determining factor in the design would be the lateral pressures generated by the heavy surcharge loads from the aggregate stockpiles.

Table 1. Soil Parameters for Bulkhead Wall Design

Soil Type	Effective Unit Wt. (γ`, psf)	Effective Friction (ϕ`, degrees)	Earth Coeff.	
			Active	Passive
Sand – Loose	110	28	0.33	N/A
Organic Silt/Clay	43	24	0.38	N/A
Sand – Medium Dense	58	32	0.28	5.8
Lean Clay – Very Stiff	68	28	0.33	4.1
Lean Clay – Stiff	68	26	0.35	3.7

N/A – Not Applicable

The operator of the facility wanted to place the aggregate piles such that the minimum distance between the toe of the piles and the bulkhead wall would be 20 feet. This would allow the freighters to offload the aggregate piles in two rows of piles directly from the ship. However, placing the piles such a short distance behind the wall generated large lateral pressures on the wall. Typically, offsets of about 50 feet from the wall to the edge of the stockpiles have been used to limit the lateral pressures on the wall from the stockpiles.

Classical methods for determining stresses in linearly elastic half space where used to determine the lateral pressure on the walls Christian and Ursua [1996]. The computer program STRESS by Christian and Ursua is based these methods and was used to estimate the lateral stress on the bulkhead wall. The conical piles were modeled as a series of circular loads with decreasingly smaller diameters stacked on top of one another. The lateral pressure distribution on the wall was then determined by adding the effect of each circle at the closest point to the wall.

Design Methods.

The author used several methods to design the walls once the lateral pressures were determined. Limit equilibrium methods were used to estimate the failure conditions for overturning and mass (slope stability). The results of these analyses provided suitable safety factors against failure, but did not provide estimates of movements at loads less than failure. Therefore, a soil structure interaction method was used with beam theory to estimate the response of the bulkhead wall under the anticipated loads.

Limit Equilibrium from Earth Pressures. Classic limit equilibrium methods based on the lateral pressures developed using log spiral earth pressure coefficients combined with the lateral pressures from the aggregate stockpiles were used to determine the size and length of the sheet-pile wall and deadman, the size and spacing of the tierods, and the size of the walers. The computer program Shoring Suite version 8 by Civiltech (www.civiltechsoftware.com) was used for this analysis.

Mass Stability. A check of the mass stability was performed for potential failure surfaces below the sheeting. The analysis was performed using the slope stability program PC-Stable, utilizing the Modified Bishop method of slices to determine the factor of safety for a series of potential failure surfaces. The analysis indicated a minimum factor of safety of 1.5 for a deep seated failure surface extending through the upper granular soils and organic silts to the top of the very stiff clay layer.

Soil-Structure Interaction. Limit equilibrium methods for earth retention design provide a good means to determine the required embedment and maximum bending moments. Slope stability analyses also provide a reasonable estimate of overall stability. However, since such methods do not include the flexibility of the walls, they do not provide accurate estimates of the lateral deflections. Therefore, a soil-structure interaction beam analysis program was used to evaluate the proposed wall system under the anticipated loading. This program models the wall as a continuous beam under defined loads, with soil resistances modeled as non-linear springs.

The active soil pressure and the surcharge pressures were applied to the wall. An elastic spring was used to model the tierod reaction. P-Y curves based on the work by Matlock [1970] and Reese et al. [1974, 1975] were used to model the non-lineal response of the soils below the cut line. The analysis results provide an estimate of the lateral deflections versus depth (see Predicted Deflection in Figure 12). The high surcharge loads resulted in relatively large lateral deflections of the sheet-piles. However, the anticipated bending moments were within acceptable limits. Since the surfaces behind the wall would not be paved, this relatively large amount of lateral movements was not a particular concern.

Verification of Design by Observational Method

Designs based on variable and complex soil parameters and conditions should be verified with methods suitable for the nature and scope of the structures. For the aggregate storage facility, the key performance requirement was the ability of the sheet-pile bulkhead wall to support the weight from the aggregate.

Fig. 6 Conical Test Piles – North Side

Since the wall is a flexible structure, the amount of deflection is not as much of a concern as the overall stability of the wall under the heavy loads from aggregate stockpiles. Therefore, just monitoring the movements at the surface would not necessarily provide the information necessary to evaluate the global stability of the wall. A series of slope indicators was chosen by the design engineer as the most effective method to monitor deep movements. The inclinometers were installed directly in front of first stockpiles of aggregate to be delivered to the site.

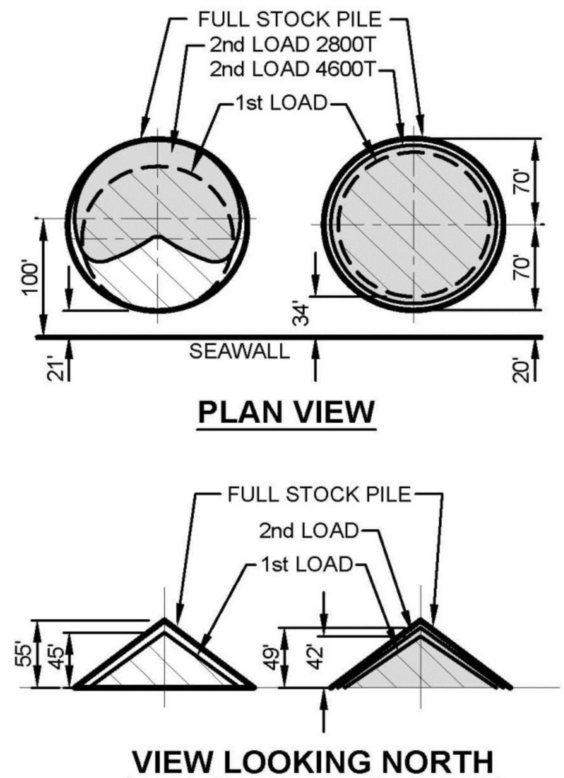

Fig. 7 Test Stockpile - North Side

The stockpiles on the north side of the slip consisted of two 16,000 ton conical shaped stockpiles of sand and gravel with a maximum height of 55 feet (see Figures 6 and 7). The stockpile on the south side of the slip consisted of a 15,000 ton elongated stock pile with a maximum height of 30 feet (see Figures 8 and 9). These stockpiles were placed in three increments over a period of about 6 weeks. The first increment consisted of a total 24,000 tons between all three stockpiles.

The second and third increments consisted of about 12,000 tons each. A delay period of about 2 weeks between placements of the aggregate loads was used to allow for an evaluation of the slope indicator information and some consolidation of the organic soils to occur.

Fig. 8 Elongated Test Pile – South Side

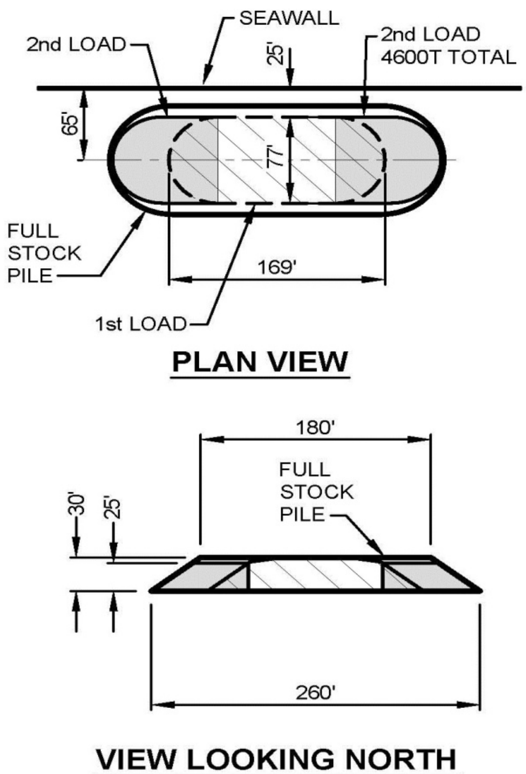

Fig. 9 Test Stockpile - South Side

Slope Indicator Results and Analysis.

A total of six (6) slope indicators were installed, with three indicators on each side of the slip. Two of the slope indicators (SI-1 and SI-3) were positioned directly in front of the conical stockpiles on the north side, with one indicator (SI-2) positioned in-between the piles. The three indicators on the south side were positioned in the middle (SI-5) and near each ends (SI-4 and SI-6) of the elongated pile.

The results of SI-1 and SI-5 are shown in Figures 10 and 11. These were the locations of maximum observed lateral movement, although the other locations showed similar magnitude and patterns of lateral movement. SI-1 on the north side shows how the lateral deflection increased as the size of the conical stockpile increased with each load increment. SI-5 at the center of the elongated pile on the south side shows that most of the movement occurred during the first load placement, since nearly all aggregate adjacent to the indicator was placed to near its maximum height with subsequent loads on each end. At all slope indicator locations, the movements stabilized within 30 days after the final placement of the aggregate test piles.

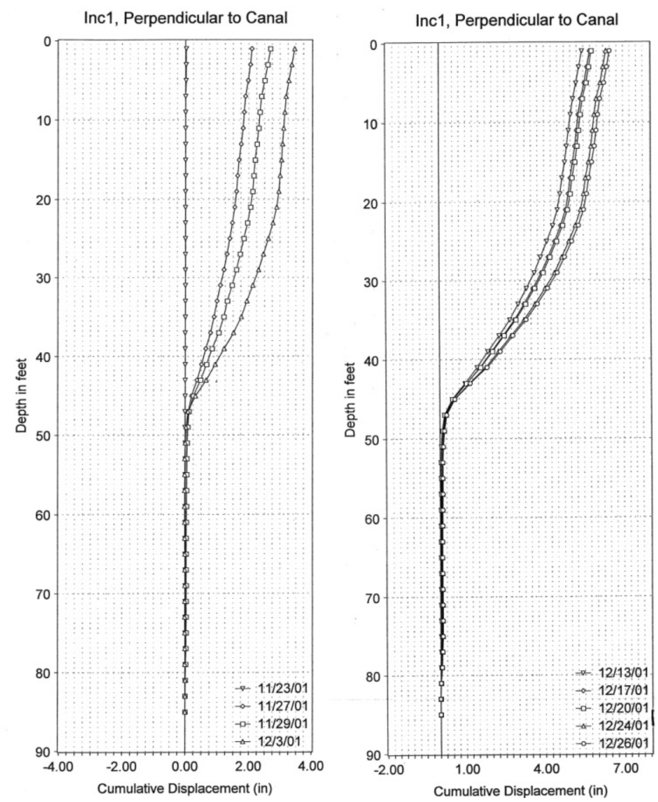

Fig. 10 Slope Indicator No 1 – North Side

Observed Deflections. As anticipated from the original soil-structure interaction analysis, there was significant deflection of the sheet-pile wall. However, the deflection was much higher at the top of the wall than originally expected. This was attributed to the extra lateral movement of the deadmen due to the surcharge loads. This "slack" movement was not accounted for in the initial soil-structure interaction analysis. When the movement of the tierods was adjusted by 2 inches, the general deflection of the wall better matches the observed maximum deflections from the slope indicators. Figure 12 shows the predicted and adjusted and deflections from the beam analysis along with the lateral deflections from the SI-1.

Bending Stresses. Note that the maximum curvature for SI-5 (and by proximity, the adjacent wall) has shifted from about a depth of 24 feet (or about 10 feet above the dredge line) to a depth of about 48 feet (or about 14 feet below the dredge line). The maximum curvature in the wall is also about where the maximum bending stresses would occur.

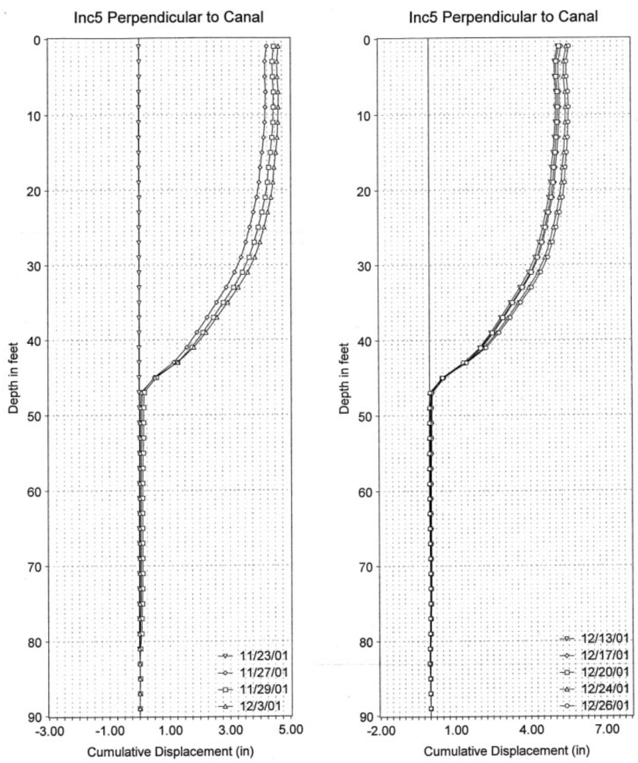

Fig. 11 Slope Indicator No 5 – South Side.

The maximum curvature point was observed near the depth to the bottom of the organic soils and top of the medium dense sand. The curvature (i.e., bending) of the sheet-piles at the interface between the organic silt and sand is judged to be due to lateral movements caused by the large stockpile loads. Although a mass stability analysis indicated a sufficient factor of safety for deep seated failure, such an analysis does not predict movements of the soil mass.

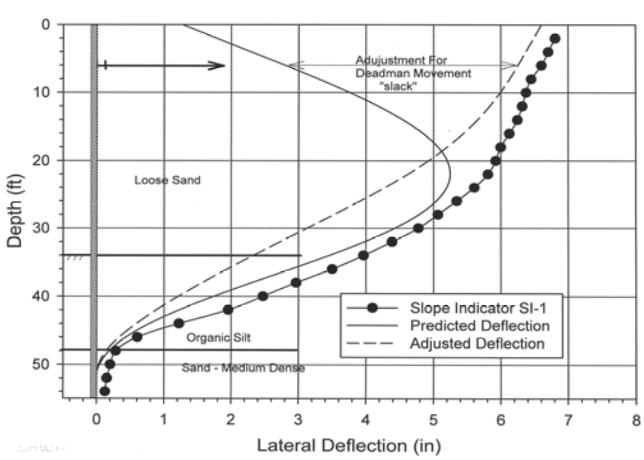

Fig. 12 Lateral Deflection of Wall (Measured and Predicted)

Bending moments in the sheet-piles can be estimated from the slope indicator data using Equation (1) which is derived from beam theory, Dunniciff [1993].

$$M_x = \frac{d\theta}{dx} EI_x \tag{1}$$

M_x = Bending Moment at Depth X
θ = Angle Measured by Inclinometer
E = Modulus of Elasticity of Steel (29,000 ksi)
I_x = Moment of Inertia at Depth X

The slope indicator data is used to determine the change in slope over a given distance (typically 24 inches for slope indicator readings), which is the first part of the term ($d\theta/dx$) in Equation (1). The change in slope over a specific distance is then multiplied by the stiffness of the sheet-pile (EI_x) to provide the estimated bending stress.

Since the data from the slope indicator is taken every 2 feet, a curve fitting method is needed to estimate the curvature of the adjacent sheet-piles. Ooi and Ramsey [1993] compared several curve fitting methods to 60 sets of inclinometer readings obtained from a variety of walls and drilled shafts instrumented with strain gauges. Based these comparisons, Ooi and Ramsey suggested using a piecewise cubic polynomial curve fitting with a moving window of five successive inclinometer data points to provide a reasonable method for estimating bending moments. This can be done fairly easily using the slope indicator readings and a computerized spread sheet.

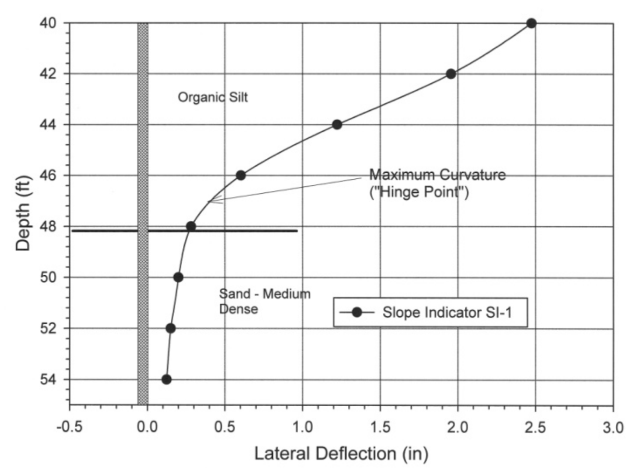

Fig. 13 Lateral Deflection Near Bottom of Wall

Figure 13 shows the inclinometer data in the area of the highest curvature near the base of the sheet-piles. Using Equation (1) and the method suggested by Ooi and Ramsey for determining the curvature of the piles from inclinometer data, a maximum bending moment of 220 kip-ft was estimated at a depth of 48 feet. This results in a maximum bending stress of 59 ksi for the JZ112 sheet-pile section (I_x=374 in^4).

The estimated maximum bending stress exceeds the yield stress of 50 ksi of the steel. Therefore, it is likely there was plastic hinge developed in the sheeting at this depth.

Overall Stability of Bulkhead. The sheet-piles not only provided a vertical wall for the docking and unloading of the freighters, but also provided additional resistance for a deep seated failure in the softer organic soils. The critical time for the stability of the wall is during the initial loading of the stockpiles. As the organic soils below the stockpiles consolidate with time, they will increase in shear strength and the overall stability should increase accordingly.

The plastic hinge in the sheet-pile wall was well below the dredge level, but above the tip of the sheets. The embedment of the sheet-piles into the sand provided sufficient passive resistance to support the bottom of the wall. Although, the upper portion of the wall moved up to 6 inches under the weight of the aggregate piles, it was held steady by the anchored deadman and the movements stabilized with time. Although a hinge developed in the sheet-piles, it did not significantly affect the performance of the wall.

CASE NO. 2: DEEP BRACED EXCAVATION

A new steel processing facility required the construction of a relatively large and deep pit. The soil conditions consisted of a deep deposit of softer clay over a dense glacial till. A heavy steel pile earth retention system was used to laterally support the excavation and to control base stability during excavation and construction. Internal steel struts consisting of pipes and steel shapes were used to internally brace the walls.

Pit Construction and Soil Conditions. The deepest portion of the pit is approximately 80 feet by 72 feet in plan area, and 33 feet deep to the base of the mat foundation. A typical profile of the pit is shown in Figure 14.

The soil conditions consist of several feet of slag fill over deep glacial lacustrine deposit of clays overlying a dense glacial till (locally referred to as ‘hardpan’). The upper 6 to 8 feet of the clay is overconsolidated due to desiccation and has a shear strength of 1,500 psf. The clay below this level is just slightly overconsolidated with an average shear strength of about 700 psf, extending to a depth of about 75 feet. The extremely dense clay till had Standard Penetration Test values in excess of 100 blows per 6 inches.

Wall Design. The depth of the pit and deep softer clay created a significant design problem related to basal stability (i.e. “bottom heave”). The shear strength of the clays was not nearly sufficient to prevent a rotational shear failure below the base of the excavation. Previous experience in the area indicated the excessive ground movements would start to occur once the excavation reached a depth of about 20 feet. Therefore, control of the bottom heave will be critical to construction of the pit.

Conventional design for deep excavations in soft clays is to extend the sheet-piles to several feet below the base to provide additional lateral resistance for basal stability.

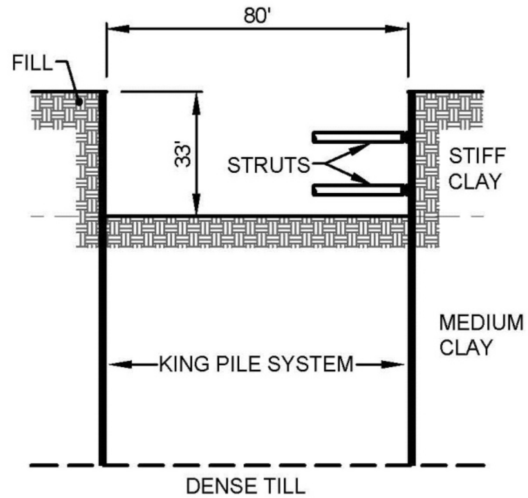

Fig. 14 King-Pile Wall Section

However, the large area of the pit extended the potential failure plane to a depth of 46 feet below the bottom of the excavation. Even using a 2/3 penetration of this depth, as is often used for such conditions, would have resulted in a minimum 30 foot embedment below the bottom of the cut. This portion of the sheeting would have been unsupported and cantilevered off the lowest strut. The length of the cantilever and the magnitude of the loads on the lower portion of the sheeting would have resulted in prohibitively high bending moments. The base of the wall was therefore extended into the dense till layer to provide lateral support at the tip. This essentially created a long beam supported at the lowest strut and at the tip.

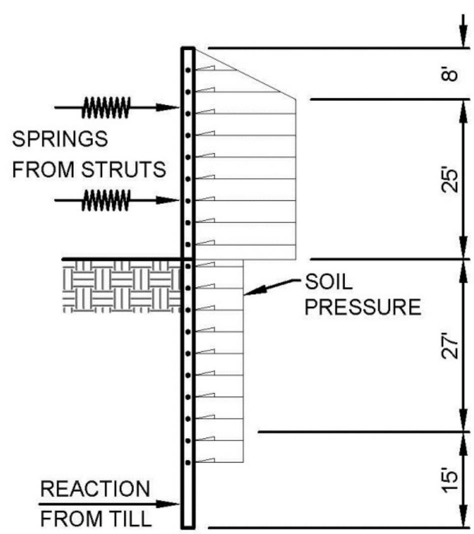

Fig. 15 Soil-Interaction Wall Model

Determination of the lateral earth pressures on the wall would be the first step in the design. For the portion of the wall above the cut, an apparent earth pressure diagram was used for soft clay, Peck et. al. [1974]. Below the cut line, an equivalent uniform pressure was estimated based on theory of basal heave develop by Terzaghi [1943]. This method determines the net force required to balance a circular shaped base failure (similar to a footing) centered around the bottom of the excavation, and extending the entire width of the excavation. This load is then applied uniformly over the portion of the wall in the zone of the failure, or a depth of about 42 feet.

To analyze the wall for moments and deflections, a soil-structure interaction beam analysis program was used to evaluate the proposed wall system under the anticipated loading.

As with the analysis used for Case History No. 1, such a program models a wall as continuous beam under defined loads with soil resistances modeled as non-linear springs Matlock [1970]. Springs were also used to model the struts based on their compressive stiffness and a reaction was set at the tip to determine the load being applied to the pile embedded into the dense till (see Figure 15).

As expected, the analysis indicated the maximum bending moments would be at a depth well below the excavation. These moments were still of such magnitude that a conventional Z-shaped sheet-pile could not be used. Therefore, a "King-Pile" system consisting of a series of wide flange sections connect to a pair of sheet-piles was used as the wall section. A W24x162 wide flange section was used for the King-Pile with a pair of PZC-18 sheet-piles in-between each wide flange section (see Figure 16). This combination resulted in the King-piles being spaced about every 5½ feet on center.

Sheet-pile connectors were welded to the flanges of the wide flange so a continuous wall can be formed with this combination. Essentially, the sheet-piles act as lagging to transfer load to the much stiffer king piles. The king piles were seated several feet into the dense till, while the sheeting was only extended through the potential failure zone.

Fig. 16 King-Pile/Sheet-pile Wall

The walls were braced at two levels with continuous walers consisting of two HP14x117 steel sections stacked on top of each other to support the relatively heavy loads. Pipe struts were used for the longer spans across the wide pit and H-pile sections were used at the corners (see Figures 17 and 18).

Verification of Design by Observational Method

Based on experience with designs, the engineer determined that the critical component of the design was the lower strut and waler system. This level would have the highest loads since it supported most of the wall, including about one-half of the soil pressure below the cut. Therefore, the performance of the wall depended on the ability of the walers and struts to resist large earth pressures caused by removing the soil from inside the excavation.

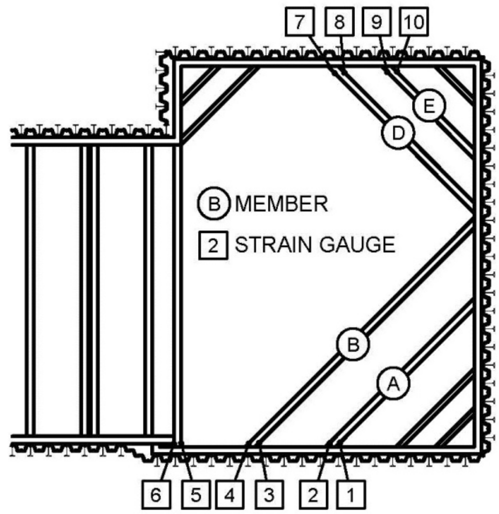

Fig. 17 Deep Pit Plan

Instrumentation. A relatively simple approach of affixing strain gauges on support elements was determined to provide the best approach for verification of the design. Two strain gauges (top and bottom) were welded to 5 members of the lower bracing system. The strain gauges (10 total) were read periodically during the excavation until the base of the excavation was reached and the concrete mat was in-place. The results from the strain gauges (in units of microstrain) were converted to each strut's load (via cross sectional area and modulus of elasticity) and then compared to the maximum allowable capacities.

Measured Loads. A maximum strut load of 508 kips was measured at location D on a 30 inch outer diameter pipe strut, with a 0.5 inch wall thickness. The maximum allowable capacity of this strut was estimated to be about 990 kips with a factor safety for 1.5 against Euler bucking. Therefore, the actual measured load was about 50% of the allowable load.

Other locations had measured loads ranging from about 22 to 51 percent of each strut's allowable capacity.

Fig. 18 Internal Bracing of King Pile Walls

The load at Location D decreased slightly to 500 kips after placement of the concrete mat against the wall. This indicates there was at least some transfer of the strut load to the mat. Eventually, the entire lower level of bracing was removed and as the base mat was used to support the wall. The upper bracing level had to remain in place until the concrete walls of the pit and the top decking placed could be constructed.

Table 2. Measurements Based on Strain Gauges

Member	Average Member Stress (ksi)	Average Member Load (kips)	Max. Allowable Load (kips) (FS=1.5)	% of Allowable Capacity
A	5.9	218.3	776	28.1%
B	4.3	197.4	890	22.2%
C	6.4	443.5	1007	44.0%
D	11.0	**508.2**	992	51.2%
E	11.3	418.4	833	50.2%

Fig. 19 Members D and E (Pipe Struts)

Comparison to Design Loads. The anticipated load on the lower waler was about 28 kips per foot based on the soil interaction beam analysis. For Member D, the linear per foot load calculates out to about 528 kips in axial load. The computed load compares within 4% of the actual measured maximum load, which is considered to be quite good.

The results of the instrumentation indicate the basic design assumptions and method were reasonable and a direct indicator that the heavy loads on the lower bracing did not overstress the members. Although, one could conclude there was significant overcapacity in the bracing, the cost of the bracing still can be justified. Most of the cost of such heavy earth support systems is in the fabrication and driving for the king piles and sheet-piles. The additional cost for heavier bracing is a relatively small cost compared to their critical nature in supporting the costly main wall.

CONCLUSIONS

Both of these case histories involved relatively complex or difficult soil conditions and large unbalanced earth pressures. However, the design for each of these systems was based on established and well known design methods. These methods used simple computer programs, but could have been done by hand if necessary. Elaborate and complex soil/structure models are not required if such methods verify, or in the case of the aggregate storage facility, recalibrate the original analysis. This is the basis for Dr. Ralph Peck's observational method and the reliance on sound judgments by the design engineers.

Interactive use of the observation by field measurements (i.e., slope indicators and strain gauges), can be facilitated by a targeted and easily managed instrumentation program. By focusing on the key design elements or areas, the amount of data to be managed and analyzed can be kept to a reasonable level. This not only makes the program more cost effective, but results in quicker and better decisions by the engineers while they are most beneficial to the construction of critical below grade structures.

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